The Association and Dissociation Tendencies of the Coaxial and Non- Coaxial Components of Shear Strength of Soils to Environment

J. Rajaraman, K.Thiruvenkatasamy and S. Narasimha Rao

***Abstract*— Soil is rock on its way to ocean. It undergoes many changes during the travel. The important factors which influence the geo-technical behavior of soil are : Grain size and Shape, Gradation, Water, Parent rock materials and Environment. The Geo-technical behavior of fine grains in soil is highly complex than coarse grains. A pure cohesive soil has pure shear or coaxial shear component only. A pure friction soil has Non-coaxial or simple shear component only. Normally a soil sample is a mixture of coarse and fine grains. The shear strength is shared between coaxial and non-coaxial component of the total shear strength of soil. The environment influences the Geo-technical behavior of soil. The coaxial and non-coaxial components of shear strength (coax and non-coax) of soil accepts and adjusts to reach a new equilibrium in stability. In this paper the association and dissociation of coax and non-coax with environmental conditions starting from laboratory and ending in marine environment through examples, illustrations and documented, reliable data available in literature.**

Keywords- Hydrolysis, Pure shear, Simple shear,Coaxial and Non-coaxial components of shear strength, Cohesion and angle of internal friction

I. INTRODUCTION

The amount of water existing in the soil mass will significantly influence the engineering behavior of soil. Karl Terzaghi has said in effect, that there would be no need for soil mechanics if not for water. This is because the presence of water affects the state of stress within a soil mass. The water content also has bearing on potential volume change, progressive failure, densification, shear strength, and settlement. The mechanism of soil –water interaction is complex and its behavior is not only dependent on soil types, but is also related to the current and past environmental conditions and stress histories, Isotropic stress and deviatoric stress

DOI: 10.5176/2335-6774_2.1.19

II. ISOTROPIC STRESS AND DEVIATORIC STRESS

Isotropic stress acts equally in all directions, it results in a *volume change* of the body. Deviatoric stress, on the other hand, changes the *shape* of a body [1]

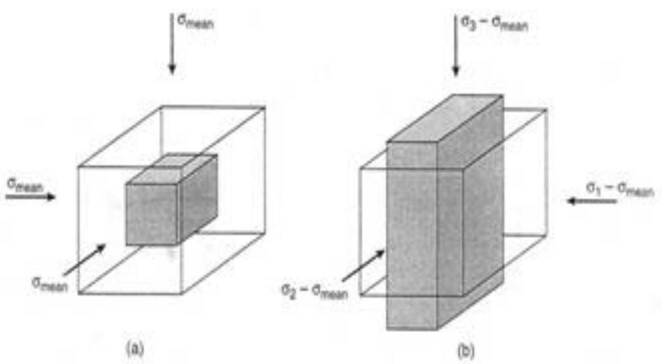

Figure 1 The mean (hydrostatic) and deviatoric components of the stress. (a) mean stress causes volume change and (b) Deviatoric stress causes shape change.

III. THE CONCEPT OF COAXIAL AND NON-COAXIAL COMPONENTS OF SHEAR STRENGTH

In a homogeneously strained, two-dimensional body there will be at least two material lines that do not rotate relative to each other, meaning that their angle remains the same before and after strain. A material line connects features, such as an array of grains, that are recognizable throughout a body’s strain history. The circle deforms and changes into an ellipse. In homogeneous strain, two orientations of material lines remain perpendicular before and after strain. These two material lines form the axes of an ellipse that is called strain ellipse. The principal incremental strain axes rotate to the finite strain axes, a scenario that is called non-coaxial strain accumulation. The case in which the same material lines remain the principal strain axes at each increment is called coaxial strain accumulation. The coaxial component of shear strength is called pure shear and the non-coaxial component of shear strength is called simple shear. The combination of simple shear (a special case of non-coaxial strain) and pure shear (coaxial strain) is called general shear or general non-coaxial strain. Two types of general shear are possible.

The following figure 2 explains simple shear and pure shear [1].

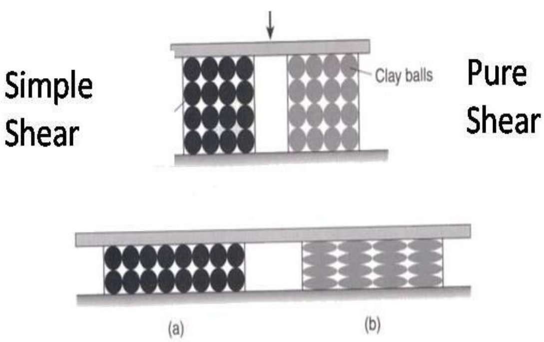

Figure. 2 Simple Shear and Pure Shear Explained

In Figure 2 the rigid spheres slide past one another to Accommodate the shape change without distortion of the individual marbles. In figure 2b the shape change is achieved by changes in the shape of individual clay balls to ellipsoids, are quite different.

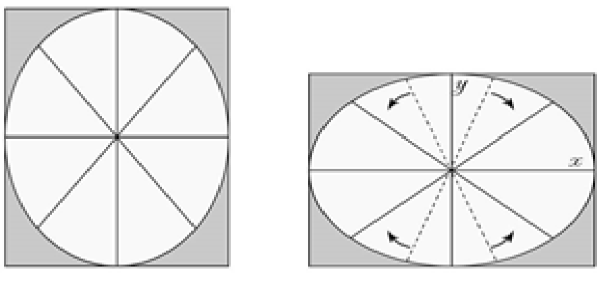

Figure.3 Homogeneous Strain

In Figure 3.Homogeneous strain describes the transformation of a square to a rectangle or a circle to an ellipse. Two material lines that remain perpendicular before and after strain are the principal axes of the strain ellipse [solid lines]. The dashed lines are material lines that do not remain perpendicular after strain; they rotate toward the long axis of the strain ellipse.

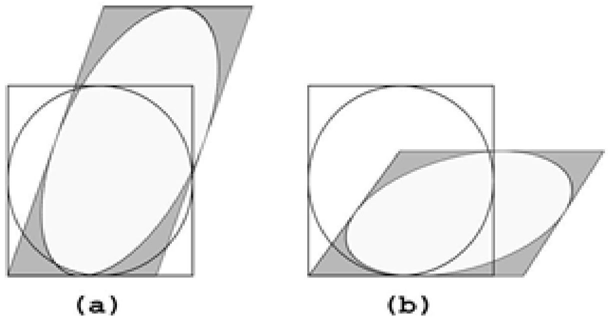

Figure.4

In Figure 4 a combination of simple shear [a special case of non-coaxial strain] and pure shear [coaxial strain]is called general shear or general non-coaxial strain. Two types of general shear are transtension [a] and transpression [b], reflecting extension and shortening components.

IV. THE COAXIAL AND NON-COAXIAL COMPONENTS OF SHEAR STRENGTH IN PROCTOR'S TEST RESULTS (LABORATORY ENVIRONMENT)

In Proctor's test the relationship between dry density and moisture content for five different soils when compacted in Proctor's mould is shown in figure 5.

The optimum moisture content corresponding to maximum dry density value for each soil is considered. For gravel- sand-clay the frictional component of the shear strength (simple shear) is relatively large and the pure shear (cohesion) is less. Similarly for sand at OMC the simple shear is more compared to the pure shear. For the last soil sample representing Heavy clay the pure shear is more and the simple shear is very much less. In other words from gravel-sand-clay to heavy clay samples the total shear strength is shared between pure shear and simple shear (coaxial and non-coaxial components of shear strength) This Proctor's test is purely between soil and water interaction. No external stress (expect compaction stress) in any form is introduced. The water content is at OMC (Optimum Moisture Content). The void is barest minimum. The cohesion and friction developed is shared by coaxial and non-coaxial components. Up to the dry density corresponding to the optimum moisture content (OMC) the association tendency is followed. When moisture content increases over and above the dissociation tendency is followed.

V. THE COAXIAL AND NON-COAXIAL COMPONENTS OF SHEAR STRENGTH IN ACTIVITY OF CLAYS (PORE – WATER ENVIRONMENT)

The direct linear relationship between PI and clay fraction content for any particular clay enables the degree of colloidal activity to be expressed very simply by the ratio:-

Activity = Plasticity index / Clay fraction.

In Geo-Technical Engineering, three classes of clay are recognized, from activity point of view, namely inactive, normal and active. For inactive clays the activity is 0.75. for normal clays the activity is from 0.75 to 1.25. For active clays the activity is 1.25. The following table gives values of PI/Clay fraction for a list of minerals. The selected minerals are Quartz, Calcite, Mica (Muscovite), Kaolinite, illite, Ca-Montmorillonite, Na-Montmorillinite. The activity of the soil gives a measure of the plasticity index of the clay fraction. There is a general correlation between activity and clay mineral compositions.

Grouping figure (5a) and (5b) side by side, can be explained in terms of coaxial and non-coaxial components of shear strength of soils. The same combined figure can be explained in terms of the influence of activities on coaxial and non-coaxial components of shear strength of clay. The clay with the higher % of clay size shows more pure shear or coaxial component of shear strength. For lower clay sized particle content in % shows less coaxial strength and frictional strength. The total shear strength is shared between coaxial and non-coaxial components of shear strength of clay. In other words the total shear strength is shared between pure shear and simple shear.

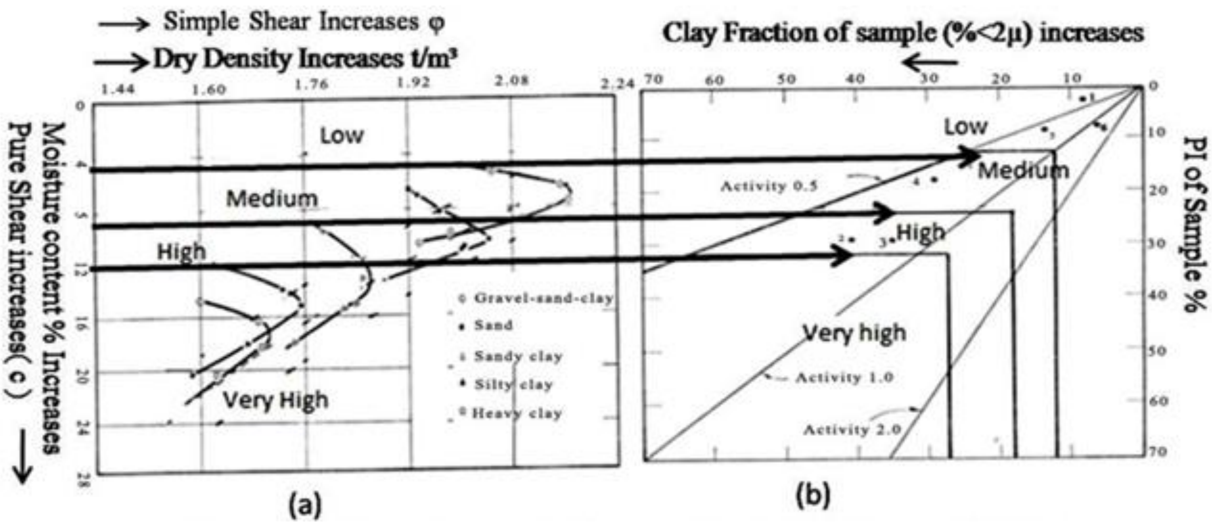

Figure 5 The Association and Dissociation Tendencies of Coaxial and Non-coaxial Components of Shear Strength in Soils.

VI. THE PROPERTIES OF SEDIMENTS DERIVED FORM DECONDARY ROCKS AND MANIFESTATION OF COAXIAL AND NON-COAXIAL COMPONENTS OF SHEAR STRENGTH

The properties of sediments derived from secondary rocks are worth mentioning in this context:

(1) Rock is aggregate of minerals. Chemical composition is a direct function of mineralogy, and mineral composition varies with grain size. The major- element chemical composition of shales and mudstones is related also to grain size.

TABLE I. VALUES OF PI/CLAY FRACTION FOR SOME CLAY MINERALS

Mineral	Activity	Reference
Quartz	0.0	Von Moos (1938)
Calcite	0.18	Von Moos (1938)
Mica (Muscovite)	0.23	Von Moos (1938)
Kaolinite	0.33 – 0.46	Northey (1950) Samuels (1950)
Illite	0.90	Northey (1950)
Ca - Montmorillonite	1.5	Samuels (1950)
Na - Montmorillonite	7.2	Samuels (1950)

(2) Grain size and shape, control coaxial and non-coaxial strains of the sediments. Angular grains increase the angle of internal -friction of the soil.

(3) Because the chemical composition of siliciclastic sedimentary rocks is closely related to the mineral composition of these rocks, the chemical composition varies as a function of grain size along with variations in mineralogy. For example that SiO_2 abundance decreases progressively from fine sands to fine clays, whereas the Al_2O_3 content systematically increases.

(4) Quartz arenites composed of 90 to 95% siliceous grains quartz, chert, quartzose rock fragments).

(5) Fine grained siliciclastic sedimentary rocks, composed mainly of particles smaller than approximately 62 microns, make up approximately 50% of all sedimentary rocks in stratigraphic record.

(6) Quartz tends to be more abundant in coarse grained mudstones and shales, whereas clay minerals are more abundant in fine grain mudstones and shales.

(7) Quartz arenites are more poorly sorted and may contain high percentages of sub-angular to angular grains. Some quartz arenites exhibit textural inversions such as a combination of poor sorting and high rounding, a lack of correlation between roundness and size, such as small round grains and larger angular grains, or mixtures of rounded and angular grains within the same size fraction. These textural inversions probably result from mixing of grains from different sources, erosion of older sandstones, or environmental variables such as wind transport of rounded grains into a quiet- water environment.

(8) Angular grains may result also from development of secondary overgrowths.

(9) Now the problem has to do with the inherent relationship of parent rock grain size and size of rock fragments. Only fine size parent rocks yield substantial quantities of rock fragments of sand size. Therefore, coarse grained parent rocks are poorly represented by rock fragments in sandstones.

(10) Collectively, the changes brought about in the composition of sediment by weathering and erosion, transport, reworking at the depositional site can be significant. Provenance analysis requires that we cannot use the absence of particular constituents as a guide to provenance interpretation; we can use only the presence. The fact that feldspars and heavy minerals may be absent or scarce in sandstone, for example, does not mean that they were necessarily absent or scarce in the source rocks. Feldspars would have been converted chemically to clays.

The ultimate products of weathering following the above properties of sediments ends up in sand and clay. The coaxial and non-coaxial components of shear strength are the hidden signature to sediments in the presence of water.

VII. THE COMPLEX FUNCTION –PERMEABILITY [2]

Permeability is a complex function of particle size, sorting, shape, packing, and orientation of sediments. These variable factors can be expressed in terms of heterogeneity factor. For a formation with a mixture of clay and sand the following equations with this heterogeneity factors C_{V_1} and C_{V_2}. This variable factor C_V is believed to decrease with decreasing particle size and decreasing sorting .This factor C_V is affected by particle orientation. It is also affected by the orientation parallel to bedding plane or perpendicular to the orientation. To make it a simple factors for the purpose of calculation the heterogeneity of clay is taken as C_{V_1} and for sand as C_{V_2}.

The general eqn for C_V total is (C_V = Coefficient of variation or Heterogeneity)

$$C_{V_{total}} = \sqrt{pc_{v_1}^2 + (1-p)pc_{v_1}^2}$$

P = 1 (Taking element No: 1 as clay)

Element 2 sand (1 - p) = o

$$C_{V_{total}} = \sqrt{1c_{v_1}^2 + (1-1)c_{v_2}^2}$$

$$\sqrt{c_{v_1}^2 = c_{v_1}} \quad \text{(for clay)}$$

Similarly for p = 0 for clay

$$C_{V_{total}} = \sqrt{0c_{v_1}^2 + (1-0)c_{v_2}^2}$$

$$C_{V_{total}} = \sqrt{c_{v_2}^2} = c_{v_2} \quad \text{(for sand)}$$

The common shear strength eqn is $\tau = [C + \sigma \tan \phi]$ where τ is shear strength, C is cohesion and σ is normal stress and ϕ is the angle of internal friction of the soil.

$$\tau = [C + \sigma \tan \phi] \cos \alpha$$

$$\cos\alpha = \sqrt{pc_{v_1}^2 + (1-p)c_{v_2}^2}$$

$$\tau = (c + \sigma\tan\phi(\sqrt{pc_{v_1}^2 + (1-p)c_2^2}$$

When α = 90^0, cos α = 0 for pure clay p = 1, Sand (1 - p) = 0, ϕ = 0.

$$\tau = (c + \sigma\tan(0))\sqrt{c_{v_1}^2 + 0c_{v_2}^2}\,,\ for\cos(90^0) = 0$$

$$\tau = (c + \sigma\tan\phi(\sqrt{c_{v_1}^2 + 0c_2^2}$$, for $\cos(90^o)=0$

$\tau = c(c_{v_1})$ for pure clay. $c_{v_1} = 1, \tau = C$

For pure sand p = 1. $\tau = (c + \sigma\tan\phi)$

$$\tau = (c + \sigma\tan\phi)\sqrt{pc_{v_1}^2 + (1-p)C_{v_2}^2}$$ for cos 0=1

For pure sand p = 0.

$$\tau = (c + \sigma\tan\phi)\sqrt{0c_{v_1}^2 + (1-0)C_{v_2}^2}$$

$$\tau = (c + \sigma\tan\phi)(\sqrt{0 + C_{v_1}^2})$$

$$\tau = (c + \sigma\tan\phi)C_{v_2}$$

For clay C = 0,

$\tau = (C_{v_2})\sigma\tan\phi$, and $C_{v_2} = 1$

Heterogeneity

C_{v_1} or $C_{v_2} = 1$

$$\tau = \sigma\tan\phi,$$

VIII. THE RELATION BETWEEN COAXIAL AND NON-COAXIAL STRAIN AND SKEMPTON POINTS

For interpretation the data (after Skempton,1964) indicating the variations of angle of internal friction (φ) with percentage of clay content is shown in a family of nine points, distributed over the first three quadrants as shown in fig.7. This figure.7, shows the sharing of coaxial and non-coaxial strain or strength by different soil samples. No point lies in quadrant IV which is high cohesion and high friction zone but in nature high cohesion and high friction cannot exist together in a soil sediment system, when sharing the same volume or space between clay and sand (0.0,1.0 or 1.0,0.0).

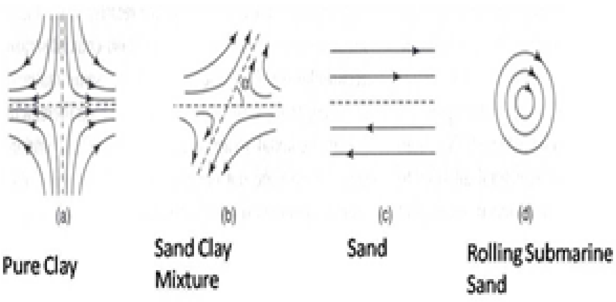

Figure.6. [3] Particle paths or flow lines during progressive strain accumulation.

These flow lines represent pure shear [a], general shear [b], simple shear [c], and rigid –body rotation [d]. The cosine of the angle α is the kinematic vorticity number,W_k for these strain histories; W_k =0,0 < W_k <1, W_k=1, and W_k = ∞ respectively. Avoiding the math, a convenient graphical way to understand this parameter is shown in fig.6. When tracking the movement of individual points within a deforming body relative to a reference line, we obtain a displacement field (or flow lines) that enables us to quantify the internal vorticity. The angular relationship between the asymptotes and the reference line defines W_k. W_k =cos α. For pure shear W_k= 0 fig. 6a, for general shear 0< W_k <1 fig. 6b and for simple shear W_k = 1 fig 6c. Rigid- body rotation or spin can also be described by the kinematic vorticity number (in this case, W_k = ∞ fig. 6d).When α = 0^0, Cos α= 1, represents simple shear. When α = 90^0, Cos α = 0, represents pure shear. [3]. The component describing the rotation of the material lines with respect to the principal strain axis is called the internal vorticity, which is a measure of the degree of non-coaxiality.

If there is zero internal vorticity, the strain history is coaxial, which is sometimes called pure shear. The non-coaxial strain history describes the case in which the distance perpendicular to the shear plane remains constant; this is also known as simple shear.

TABLE II ACTIVITY NUMBER, CLAYS AND SKEMPTON POINTS [4] [5]

Quadrant	No. of Skempton Points	Type of Quadrant
I	02.	Low cohesion, High friction
II	03.	Low cohesion, Low friction
III	04.	High cohesion, Low friction
IV	nil	High cohesion, High friction

In the fig.7.,if ∝ = 0, the slope line coincides with x axis , GFE, cos ∝= cos0 = 1.0. If ∝ = 900, the slope line becomes vertical and coincides with y axis, GHA, cos∝ = cos 900 = 0.0.

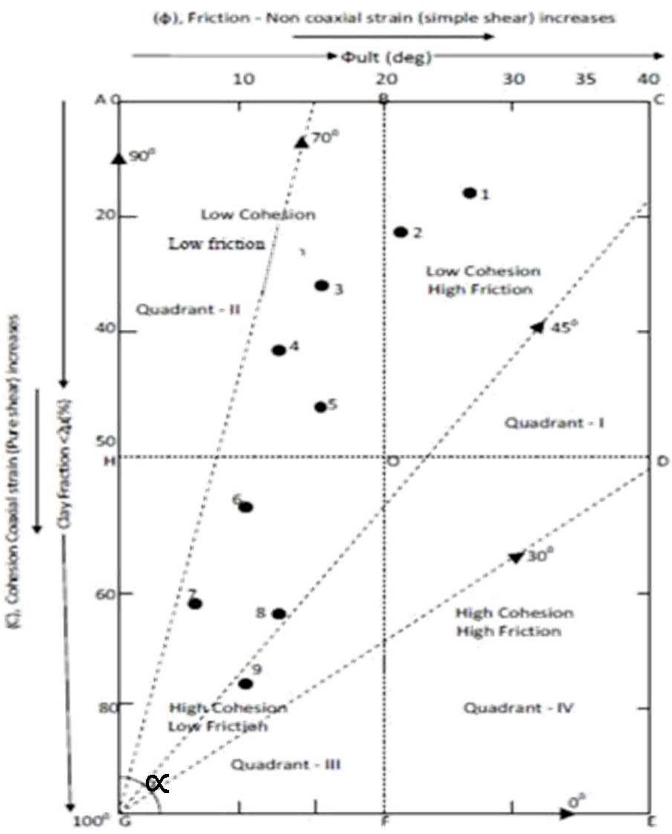

Figure.7 Variation of φ_{ult} with percentage of clay content. (After Skempton 1964) All Skempton points lie in quadrants I, II,III. [4] [5]

In Table II the activity increase from quartz to clay minerals or from frictional soil to cohesive soil or simple shear to pure shear combinations as Skempton Points is shown.

Coaxial & Non Coaxial (Pure & Simple shear) Strain Combinations

Table III is extracted from figure 8 taking into consideration soil mixture types, symbols and also whether they are active coaxial type or pro-coaxial type or active non-coaxial type.

A. Soil type : Bentonite --- Sand

As the percentage of Bentonite clay fraction increases the coaxial component of shear strength (c) increases and the residual angle of internal friction (φ) decreases. Since Bentonite is active coaxial type, the residual φ falls rapidly and the coaxial component of shear (pure shear) dominates.

B. Soil type : Kaolin --- Sand

As the percentage of Kaolin clay fraction increases the pro- coaxial component of shear strength (c) increases and the residual angle of internal friction φ decreases. Since Kaolin is only pro-active coaxial type, the residual friction φ falls slowly and at the end slowly the coaxial component of shear (pure shear) dominates.

C. Soil type : Bentonite --- Kaolin

As the percentage of Bentonite – Kaolin clay fraction increases the coaxial component of shear strength (c) dominates and the residual friction angle φ is the residual

D. Soil type : Bentonite --- Kaolin ---- Sand

As the percentage of Bentonite – Kaolin clay fraction increases the coaxial components of shear strength (c) increases but the sand introduces frictional or non- coaxial component of shear strength (Φ) and ends up in low residual friction . Since natural soil samples randomly contain clay – sand % the points are scattered and prediction becomes complex.

TABLE III GEO –TECHNICAL BEHAVIOUR OF SAND – CLAY MIXTURES [6] [7] [8]

Association and Dissociation of Coaxial and non-coaxial components of Shear strength manifestation with soil types					
Sl.No	**Soil Type**	**Symbol**	**Active Coaxial Type**	**Pro coaxial type**	**Active Non-Coaxial Type**
1	Bentonite-Sand	⊠	Bentonite	———	Sand
2	Kaolin-Sand	■	———	Kaolin	Sand
3	Bentonite-Kaolin	●	Bentonite	Kaolin	———
4	Bentonite-Kaolin-Sand	▲	Bentonite	Kaolin	Sand
5	Natural samples	◆	←	All Possible combinations	→

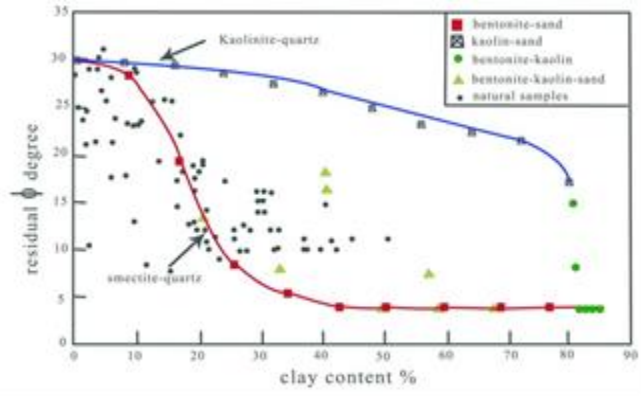

Figure 8. Residual friction, clay content % and soil types

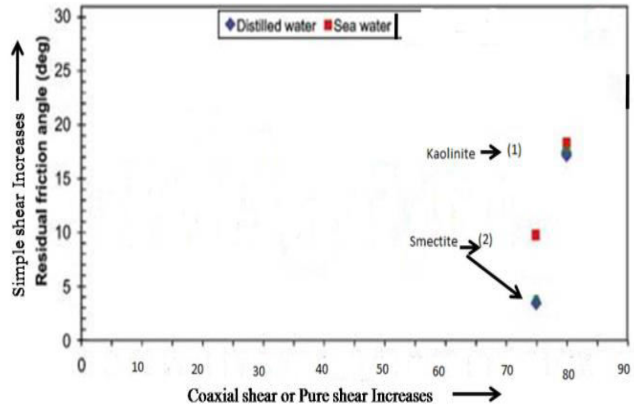

Figure 9.Variations of simple shear or pure shear with distilled water, Sea water [6] [7] [8]

Besides mineralogical composition, pore water chemistry places an important role in the residual shear strength of sediments. The residual strength of different minerals were measured with sea water as a pore fluid, Smectite showed significant increase in residual friction angle (increased to 100 from the value of 40 with distilled water) as shown in Fig.9. Kalonite did not show much influence of sea water on residual shear strength. The coastal environment (sea water) influences shear strength. Solid water in diffused double layer is responsible for resistance in Smectite which is absent in Kaolinite.

IX. DISTILLED WATER, NACL SOLUTION AND SOIL BEHAVIOUR

The weathering and stress history of the continental Rock transported as sediment as varying stages to abyssal plain meets varying or different environmental conditions on the way. Any Experimental data collected on Continental areas should be viewed with environmental information as an additional parameter to interpret Marine conditions. A few documented cases are discussed below.

A. Documented Cases

I) Case (i) Effect of Distilled Water and nacl Solution on Liquid Limit of Soil Samples

The liquidity limit obtained with distilled water is about two times the one obtained with Nacl solution, regardless its molarity.A soil with a higher liquid limit will settle more. Distilled water is not a suitable pore liquid because it helps to obtain more settlement. In this context saline water is a better pore fluid compared to distilled water. The salinity of fresh water is in between distilled and Saline Sea water.

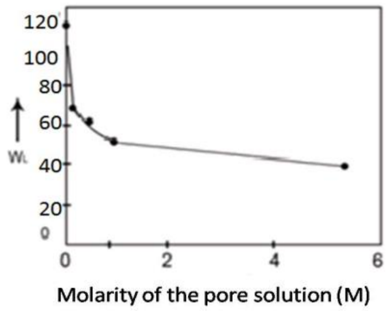

Figure. 10 Variations of liquid limit value with pore water types [9]

II) Case (ii) Odometer Test with Distilled Water and NaCl Solution of Soil Samples

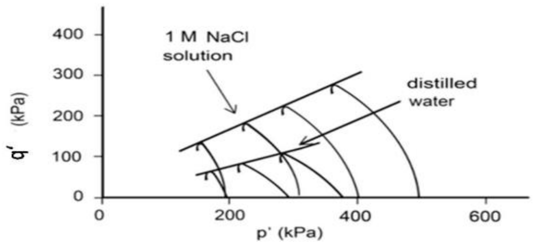

Figure.11 Odometer test with distilled water and NaCl [9] [10]

An example of possible effects of infiltration of fresh water in a natural deposit subjected to swelling is shown in fig 11. The results of an Odometer tests carried out on two couples of undisturbed specimens of a type of clay shale taken respectively at a depth of 2.5 and 21m. A specimen of each couple was tested in a 1 m NaCl solution. The influence of the nature of the bath does not appear significant in the stage of compression, when the pore water is expelled from the specimen, but becomes prominent in the following stages of swelling, when some liquid is absorbed from the bath. The specimens tested in the distilled water (and specially the one taken at the greatest depth) display higher strains than those tested in the solution. At the end of the tests performed in the NaCl solutions, when the axial stress was 10 kPa, the solution was substituted with distilled water giving immediately rise to further strong soil deformation.

III) Case (iii) Triaxial Test with Distilled Water and NaCl Solution on Soil Samples

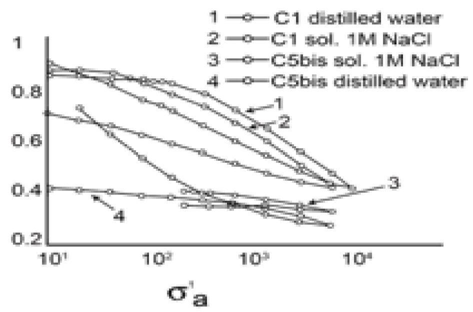

Figure 12a Triaxial test with distilled water and NaCl solution [9] [10]

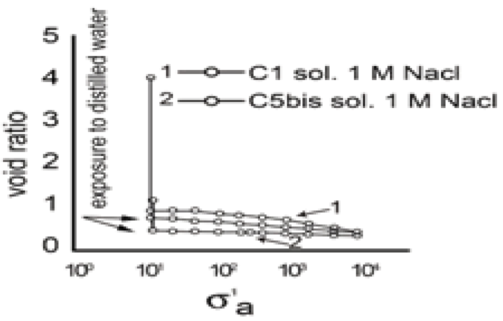

Figure. 12b Conventional test results with distilled water and NaCl solution on soil samples [9] [10]

Fig (12a, 12b) shows the results of a triaxial tests on reconstructed normally consolidated specimen obtained by mixing powdered clay with distilled water and a 1M NaCl solutions. It suggests that that a friction angle at constant volume (i.e. the critical friction angle) can strongly depend on the nature of the liquid. Similar data feature the residual friction angle.

IV) Case (iv) Swelling Test on Soil Samples

Figure 13 shows the results of conventional tests conducted under a normal stress lower than swelling pressure (around 0.6 M Pa) while part of specimens were allowed to swell, as usual, for 48h,others were sheared only after 10-100 days during which they experienced a high volumetric strain. The difference in shear strength is very clear. All data show that swelling in distilled water as usual in laboratory, is responsible for a radical change in soil behavior. In addition fig 13 shows that the effect of swelling is higher if secondary swelling is allowed.

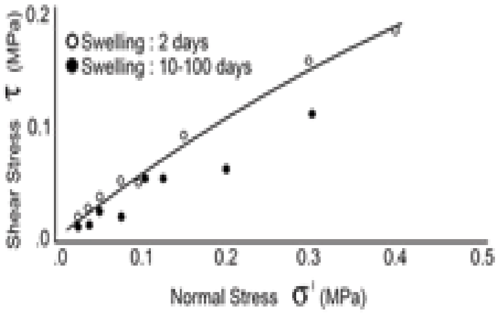

Figure.13 Swelling tests [9] [10]

X. CONCLUSIONS

The association and dissociation tendencies of the coaxial and non-coaxial components of shear strength of soils / sediments are illustrated with examples from available data in literature in the following environments:

(a). Laboratory level compaction of different types of soils ranging from gravel to clay size.

(b). The environment related to activity of clays.

(c). Different clay types from Skempton points.

(d). Heterogeneity of sediments in marine environment.

(e). Prepared clay types and sand – clay mixtures.

(f). Invisible environmental connecting threats are the strongest ties.

REFERENCES

[1] Van der pluijm,ben a, Earth Structure –W.W.Norton & Company inc, New York,10110 , 2003, pp. 65 - 68

[2] Jensen Jerry, Statistics for petroleum engineers and geoscientists,Prentice Hall, Cuevas 1993, pp. 32 -36, chapter 6 pp. 143-150

[3] Van der pluijm,ben a, Earth Structure –W.W.Norton & Company inc, New York,10110 , 2003, pp. 68

[4] Skempton, a.w, long term stability of clay slopes, Geotechnique,vol.14, 1964,pp.77

[5] Rajaraman J, "The Intrinsic Relationship among coaxial and non-coaxial strain , Bowen's series of minerals and textural maturity of sediments in lowlands" J. GSTF-International Journal of Geological Sciences, Vol. 1 April 2013, pp. 30-37

[6] Tiwari B, Marui H, "A new method for the correlation of residual shear strength of the soil with mineralogical composition" J.Geotech Geoenviron, 131 (chap 9) 2005, pp. 1139 –1150.

[7] Tiwari B, Marui H. " Estimation of residual shear strength of Toyoura sand – Bentonite – Kaolin mixture " J. Jpn Landslide Soc. 40 (2) 2003,pp. 124—133.

[8] Mitchell JK, Saga K, Fundamental of Soil Behaviour, 3rd ed. John Wiley & Sons inc, U.S.A.

[9] Di Maio C, Onorati R (2000). Influence of pore liquid composition on the shear strength of an active clay In: Bromhead E. N Dixon and M.L Ibsen (eds) Proceedings. 8th International .Symposium on Landslides, Cardiff 1: pp.463-468.

[10] Picarelli L, Di Maio C.Olivares L, Urciuoli G (1998). Properties and behavior of tectonised Clay Shales in Italy In: Evengelista A and L Picarelli (eds). Balkema Proceedings Second International Symposium on The Geotechnics of Hard Soils, Soil Rocks, Napoli 3, pp. 1211-1241.

AUTHORS' PROFILE

Dr. J. Rajaraman, M.Sc Tech, M.S. Ocean Engineering , IIT Madras is a Professor in the Department of Harbor and Ocean Engineering in AMET University, India. He has more than thirty years of teaching and research experience. His research interests include Marine Geo-Technical Engineering, Marine Geology and Marine Environmental Geo-Technology. He has recently published more than 20 research articles in reputed International journals/ Conferences held at India, Japan (Yokohama, Tsu City, Tokyo and Nagoya), Thailand, Malaysia, Singapore, Dubai and Australia. He is a senior member of the APCBEES. He is in the Technical Committee of ICESD 2015..

Dr. K.Thiruvenkataswamy, is a life member of ISTE India, Editorial Advisory Panel Member of ICE Journal of Energy, UK. A graduate Civil Engineer, holding a Doctoral Degree (Dr. Eng.) from Kagoshima University, Japan, completed Master's in Ocean Engineering from IITM. He has published several papers in International Journals; participated in International Conferences in UK, USA, Japan, Canada, Belgium, etc., His main field of research is on hydrodynamics of wave power devices.

Dr. S.Narasimha Rao was a faculty member at Indian Institute of Technology, Madras at Chennai in India during 1974-2006.He guided 24 Ph.D.theses and over 80 post graduate students in the field of marine geotechnical engineering and coastal engineering. He had published over 220 research papers in leading international and national journals and conferences .He was involved in several international and national projects in the field of Port and Harbours and several high rise structures .He was involved in teaching in a few universities in India and abroad.